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REPLY TO:

LAND & WATER MANAGEMENT DIVISION
PO BOX 30458
LANSING MI 48909-7958

November 13, 2001

TO: Janice Tompkins, Nonpoint Source Unit
Surface Water Quality Division, Grand Rapids District Office

FROM: Dave Fongers, Hydrologic Studies Unit
Land and Water Management Division

SUBJECT Blakeslee Creek Model Calibration

As requested, the Hydrologic Studies Unit (HSU) of the Land and Water Management Division (LWMD) has completed its calibration of the Blakeslee Creek hydrologic model. This analysis was requested in support of a Section 319 grant that is intended to develop a design to rehabilitate the tributary through a Clean Michigan Initiative (CMI) grant. Nothing in this report is an authorization to do any work within the watershed that would require a permit or guarantees that work proposed based on this report will be permitted or funded.

Preliminary results from this model were discussed in our report dated September 6, 2000. To assist in improving that model, watershed monitoring data were collected from April 10 to June 18, 2001, and were released on May 9 and July 17, 2001. This report discusses the refinements of the model based on additional information and the calibration of the model to the monitoring data. Monitoring data used in the calibration process is included in this report, but the entire monitoring dataset is not.

This report highlights the changes made to the model and the revised results. Appendices A, B, and C are attached which detail the basis for the hydrologic characteristics that were incorporated in the model, the calibration process, and the final model parameters respectively.

Summary

The Blakeslee Creek hydrologic model was extensively revised to include the detention ponds and to refine other hydrologic parameters based on additional information, including calibration data from monitoring water elevations at five locations within the watershed.

As a result of this effort, we have concluded that the two main detention ponds, designed to control the 4 percent flows, are attenuating smaller flows better than expected. This is because:

- the larger detention pond north of the stream has 142 percent more storage capacity than designed or required by Kent County standards
- the detention pond south of the stream has a drop structure that provides additional restriction to flow
- the function of the detention pond outlets at lower flows are better described using the more restrictive inlet control equations, rather than the outlet control equations that were used for their design

The additional restriction to flow in the south detention pond, combined with the loss of flood storage capacity due to the added one foot riser pipe, does mean that the model predicts the capacity of the pond and spillway will be exceeded during a one-percent chance (one-year), 24-hour storm, and the berm will be overtopped. This prediction does not take into account the more recent extension of the riser pipe, which further reduces flood storage capacity. The north detention pond is not at risk because of its extra volume.

Although the detention ponds are attenuating the smaller flows, the flow regime of Blakeslee Creek has been significantly altered. Runoff volumes and channel-forming flows have increased. Channel-forming flow is the flow that is most effective at shaping the channel. In a stable stream, the channel forming flow has a one to two year recurrence interval and is the bankfull flow. Runoff volumes to the stream just below the subdivision are projected to increase by 167 percent for the 50 percent chance 24-hour storm, to 30 percent for the 1 percent chance 24-hour storm. The 50 percent peak flow is projected to increase by 70 percent from 2.3 to 3.9 cubic feet per second (cfs). The stream channel will also be exposed to higher flows for a longer time. The combination of increased channel forming peak flow and extended duration of higher flows is morphologically destabilizing. The extensive erosion and downcutting we have observed in the channel are symptoms of this instability.

Model Refinements

In our preliminary report, we recommended that the watershed delineation and land use boundaries be verified and that detention be added to the model. As a result of our meeting with the developer's engineer, the watershed delineation in the preliminary report is considered valid, but we have increased the extent of single family residential land use in the middle subbasin from 47.1 acres to 57.7 acres.

We have added detention to the middle subbasin. Because there are three detention ponds, the middle subbasin was revised to have four drainage areas, one for each area draining to a detention pond and one for the rest of the subbasin. The detention ponds were modeled based on the provided engineering material, which was adjusted as necessary based on field information. In particular, the design of the outlet from the south detention basin was altered to include the one foot riser pipe that was not part of the original design. This pipe removes one foot of useable flood storage and, compared to the unmodified design calculations, alters the discharge rate from the detention pond until the riser pipe is fully submerged. A more recent extension of the riser pipe is not included in the model. That modification is apparently temporary until the engineering firm developing a rehabilitation design for the stream can provide its recommendations for the detention pond.

Using the revised land uses and drainage areas, we recalculated the subbasin runoff curve numbers. This highlighted a problem with our GIS-based system of calculating curve numbers. This system defaults single family residential land use to a lot size of 1/8 acre or less. The average lot size in the middle subbasin is approximately ½ acre. The curve numbers for single family residential are significantly lower for larger lot sizes. The calculated curve numbers of 75 and 72 for the middle and lower subbasins in the preliminary report, which have areas of single family residential land use, are too high. The curve numbers calculated for this report and shown in Table 1 were adjusted manually.

Table 1 - Calculated Curve Numbers

Subbasin	Area (square miles)	Runoff Curve Number	
		1978	Current
Upper (above subdivision)	0.145	51*	51*
Middle (vicinity of subdivision)	0.119	59	68
Lower (mouth to subdivision)	0.058	67	67

**the curve number after model calibration is 46.6*

Other changes to the model include:

1. The storage coefficient was changed from 0.6 times the time of concentration to 1.0 times the time of concentration. Research has indicated that this better replicates average Michigan conditions.
2. The initial loss was changed to the equation available in Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) that is based on the curve number, rather than a fixed 0.5 inches.
3. The reach routing method was changed to the Modified Puls method. The values selected were based on HEC-River Analysis System (HEC-RAS) modeling and the monitoring data.
4. The soils that were classified as B/D were reclassified as B because the soils were near the stream and, therefore, can be considered drained.

Further refinements were made based on the calibration data as discussed in Appendix B. The calibration data served to highlight areas where the analysis was incomplete or in error. The modeled north detention pond volume, for example, was initially based on the values from the consultant's engineering analysis. Based on the monitoring data, the modeled pond's volume appeared to be too small. The calculated volume based on the engineering plans is significantly larger and reproduces the monitoring data better.

Results

The results indicate a 175, 94, 69, and 49 percent increase in runoff volume from the subdivision for 50, 10, 4, and 1 percent chance (2-year, 10-year, 25-year, and 100-year) 24-hour storms, respectively, as shown in Table 2. Peak flows from the 50 percent chance 24-hour storm are predicted to be 70 percent higher. Peak flows from the 10 and 4 percent chance 24-hour storms are predicted to decrease by 58 and 23 percent, respectively. Peak flows from the 1 percent chance 24-hour storm are approximately equal. The overflow spillway capacity was exceeded during this storm, adding uncertainty to the results. Similar increases are predicted for the stream below the middle subbasin, as shown in Table 3 and Figures 1 through 8, which demonstrates the dominance of the runoff contribution from the middle subbasin to the total runoff volume and peak flow. The results in Table 2 are for runoff from the middle subbasin only; the results in Table 3 and Figures 1 through 8 are flows in the stream and include flows from the upper watershed.

Smaller relative increases in runoff volume are predicted for the stream at the mouth. As with the middle subbasin, the model predicts an increase in peak flows from the 50 percent storm and decreases from the 10 and 4 percent storms. These results assume the wetland in the lower subbasin continues to function to moderate the flows, an assumption which is in jeopardy.

These results are significant whether considering bridge replacement, reestablishment of a cold water trout stream, or stabilization of the stream morphology.

Table 2 - Selected Model Results for Middle Subbasin

Modeled Discharge Contribution from Middle Subbasin					
24-hour Rainfall	Land Use Scenario	Peak Discharge (cfs)	Percent Change	Discharge Volume (acre-feet)	Percent Change
50% chance	1978	2.3	70	0.8	175
	Current	3.9		2.2	
10% chance	1978	18.3	-58	3.1	94
	Current	7.7		6.0	
4% chance	1978	43.6	-23	5.9	69
	Current	33.4		10.0	
1% chance	1978	103	2*	12.3	49
	Current	105*		18.3	

**Value is approximate because the model predicts the south detention basin will exceed capacity.*

Table 3 - Model Results for Blakeslee Creek below Middle Subbasin

Modeled Discharge in creek below Middle Subbasin					
24-hour Rainfall	Land Use Scenario	Peak Discharge (cfs)	Percent Change	Discharge Volume (acre-feet)	Percent Change
50% chance	1978	2.3	70	0.9	167
	Current	3.9		2.4	
10% chance	1978	18.4	-55	4.1	71
	Current	8.2		7.0	
4% chance	1978	43.9	-16	8.7	47
	Current	36.9		12.8	
1% chance	1978	106	10*	19.9	30
	Current	117*		25.9	

**Value is approximate because the model predicts the south detention basin will exceed capacity.*

Table 4 - Selected Model Results for Blakeslee Creek at Mouth

Modeled Discharge in creek at Mouth					
24-hour Rainfall	Land Use Scenario	Peak Discharge (cfs)	Percent Change	Discharge Volume (acre-feet)	Percent Change
50% chance	1978	4.4	41	3.0	50
	Current	6.2		4.5	
10% chance	1978	19.7	-23	7.9	37
	Current	15.1		10.8	
4% chance	1978	55.3	-12	14.3	29
	Current	48.9		18.4	
1% chance	1978	121	4*	29.3	20
	Current	126*		35.3	

**Value is approximate because the model predicts the south detention basin will exceed capacity.*

If you have any questions or comments regarding our evaluation, please contact me at 517-373-0210.

Attachments

cc: Nicole Stout, GVSU Water Resources Institute
Michael Young
Daniel Melpolder
Ralph Reznick, SWQD
Ric Sorrell, LWMD
Barry Horney, LWMD
Dave Price, LWMD
Robert Day, LWMD

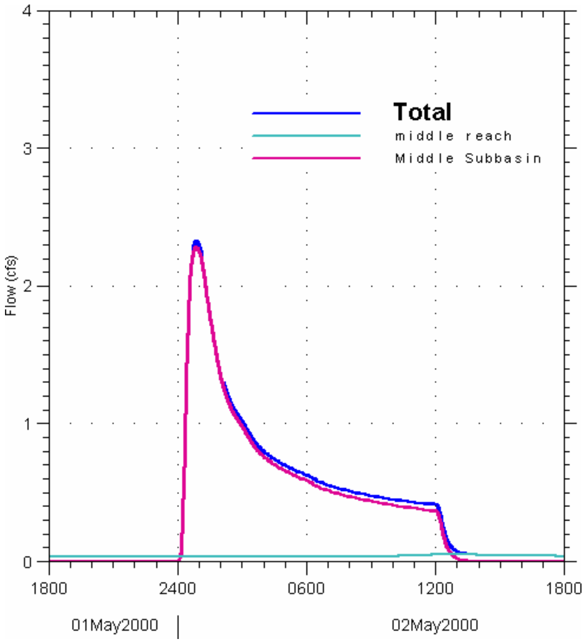

Figure 1 - Model Results: 50 Percent Chance 24-hour Storm, Pre-Development, Below Middle Subbasin

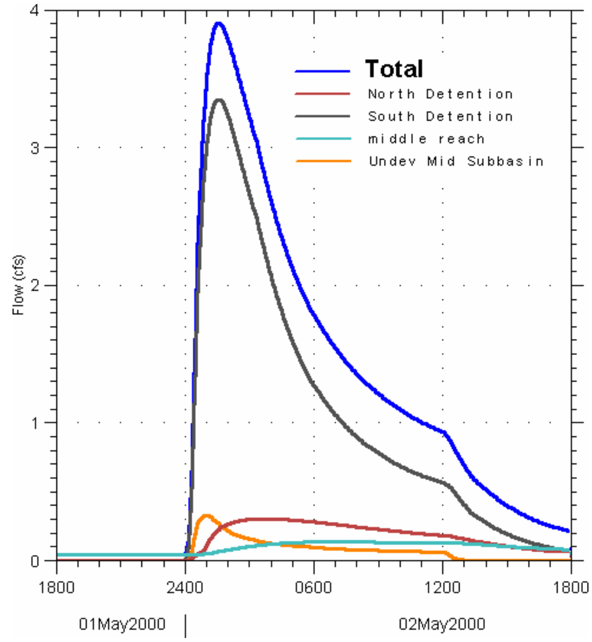

Figure 2 - Model Results: 50 Percent Chance 24-hour Storm, Current Development, Below Middle Subbasin

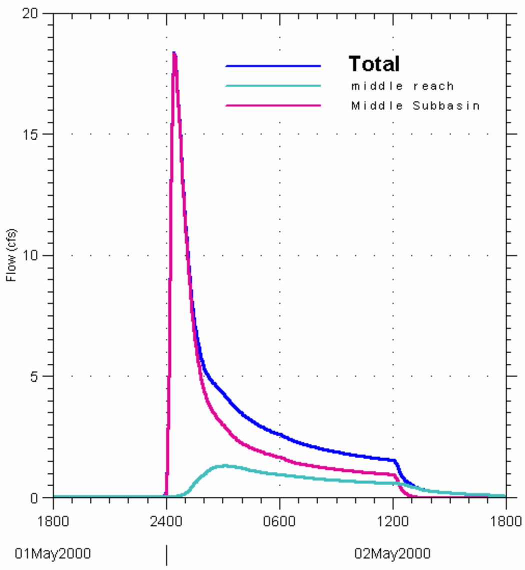

Figure 3 - Model Results: 10 Percent Chance 24-hour Storm, Pre-Development, Below Middle Subbasin

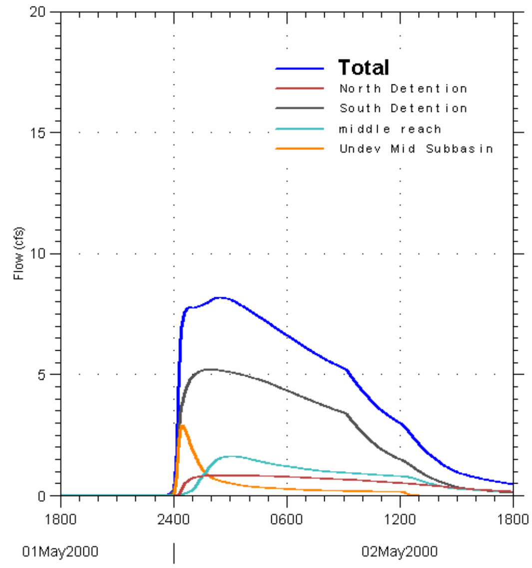

Figure 4 - Model Results: 10 Percent Chance 24-hour Storm, Current Development, Below Middle Subbasin

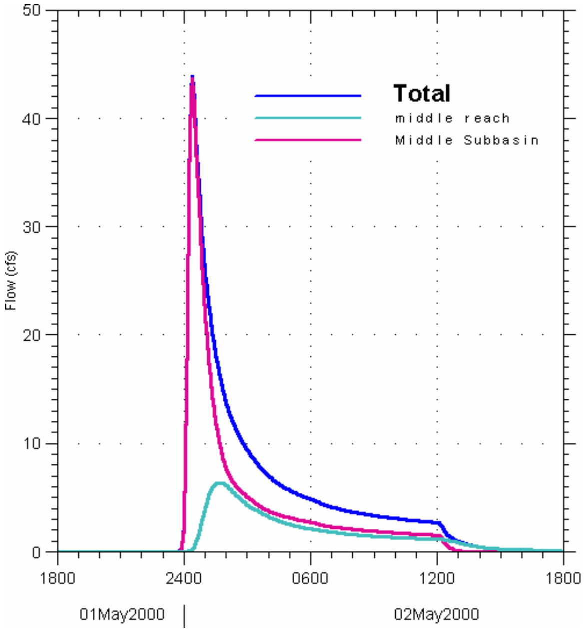

Figure 5 - Model Results: 4 Percent Chance 24-hour Storm, Pre-Development, Below Middle Subbasin

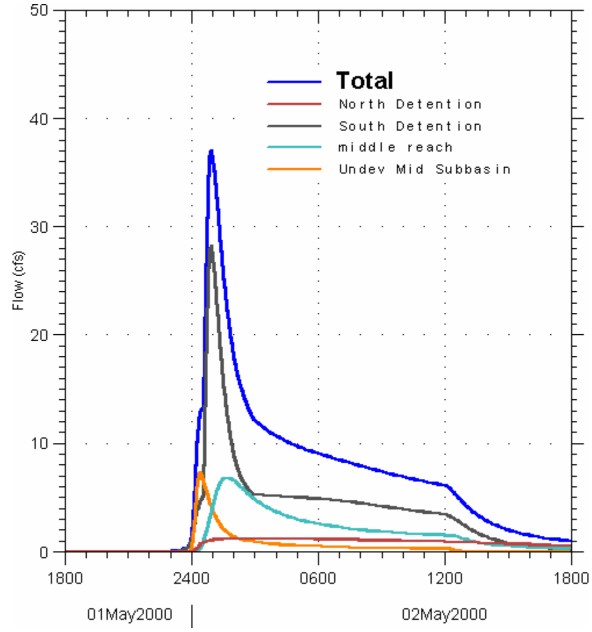

Figure 6 - Model Results: 4 Percent Chance 24-hour Storm, Current Development, Below Middle Subbasin

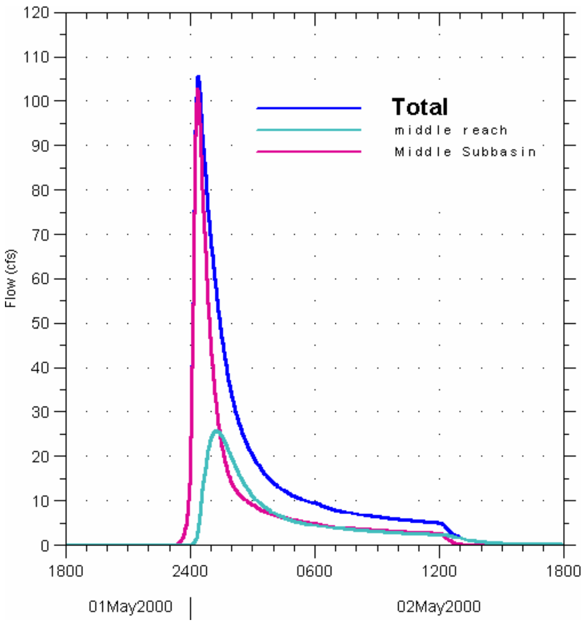

Figure 7 - Model Results: 1 Percent Chance 24-hour Storm, Pre-Development, Below Middle Subbasin

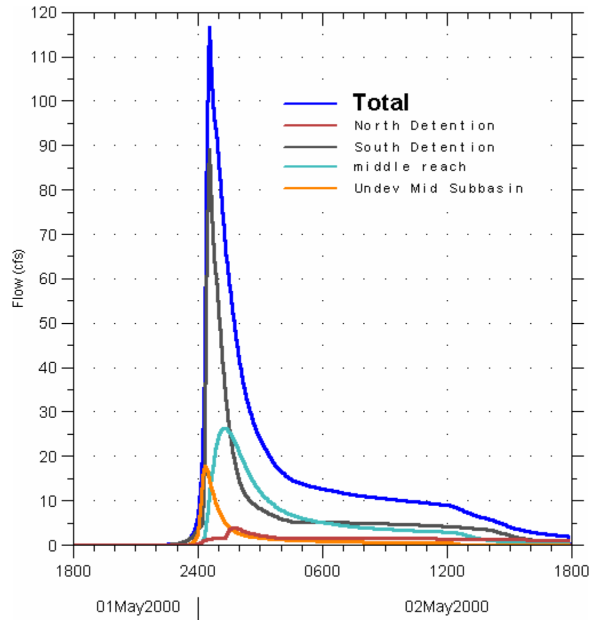

Figure 8 - Model Results: 1 Percent Chance 24-hour Storm, Current Development, Below Middle Subbasin

Appendix A
Determination of the Hydrologic Characteristics Of Blakeslee Creek Watershed

This watershed study was initiated because of the large volume of sediment moving through the stream into the Rogue River, a designated trout stream. The sediment was caused by poor erosion control practices during the development of a subdivision in the middle portion of the watershed and by streambank and streambed erosion of the stream channel. The channel may have become morphologically unstable because of increased runoff from the subdivision. Morphologic instability is characterized by extensive, accelerated channel erosion.

The goal of this study is to better understand the watershed's hydrology so that:

- a suitable, long-term rehabilitation BMPs can be selected and designed
- the impact of the rehabilitation designs can be predicted
- further changes in the flow regime of Blakeslee Creek due to future hydrologic changes within the watershed can be predicted and controlled appropriately

The delineated watershed area is 0.32 square miles as shown in Figure 1. As shown in the July 1992 aerial photo, Figure 2, the watershed then was primarily natural areas, with a small amount of residential land use. Significant earthwork is visible in the spring 1998 aerial photo, Figure 3.

Pre- and post-development runoff curve numbers were calculated from soil data and 1978 or current land use data shown in Figures 4 through 6, respectively. Estimated current land use is identical to the 1978 land use except that the middle subbasin was redefined as predominately single family residential. Initial estimates of time of concentration were calculated from the USGS quadrangles.

These parameters were then incorporated into a model using HEC-HMS. The model computes runoff volume and flow. The modeled precipitation events were the 50, 10, 4, and 1 percent chance 24-hour storms. Design rainfall values for these events are tabulated in *Rainfall Frequency Atlas of the Midwest*, Bulletin 71, Midwestern Climate Center, 1992, pp. 126-129, and summarized for this site in Appendix C. Figure 7 shows the hydrologic elements in the model for the 1978 land use simulation. In order to incorporate detention for the residential development in the middle subbasin, the model for the current land use simulation uses four, rather than one, drainage areas to represent the middle subbasin, as shown in Figure 8.

The storage-discharge relationships for the detention ponds were based on standard culvert equations. Our analysis included calculation of the culvert capacity when the culvert is functioning under inlet and outlet control. The equation that predicts the lesser flow for a given water depth, or head, would govern. The south detention includes a calculation for the riser that has been added. We do not have an equation for a device of this type that does not include vortex control. We assume that the weir equation which we have used would over-predict flow because the converging flow paths into the riser would reduce the effective weir length and because the vortex generated until the riser is fully submerged further reduces flow. We reduced the diameter term in the equation by half to account for these losses. The storage volumes are based on the engineering plans where there is a discrepancy with the volumes used in the calculations provided by the consultant. Figures 9 through 12 show the results for the north and south detention ponds, respectively. The northwest detention pond's values for storage and discharge were assumed equal to 36.8 percent of the values for the north detention pond, based on the ratio of drainage areas.

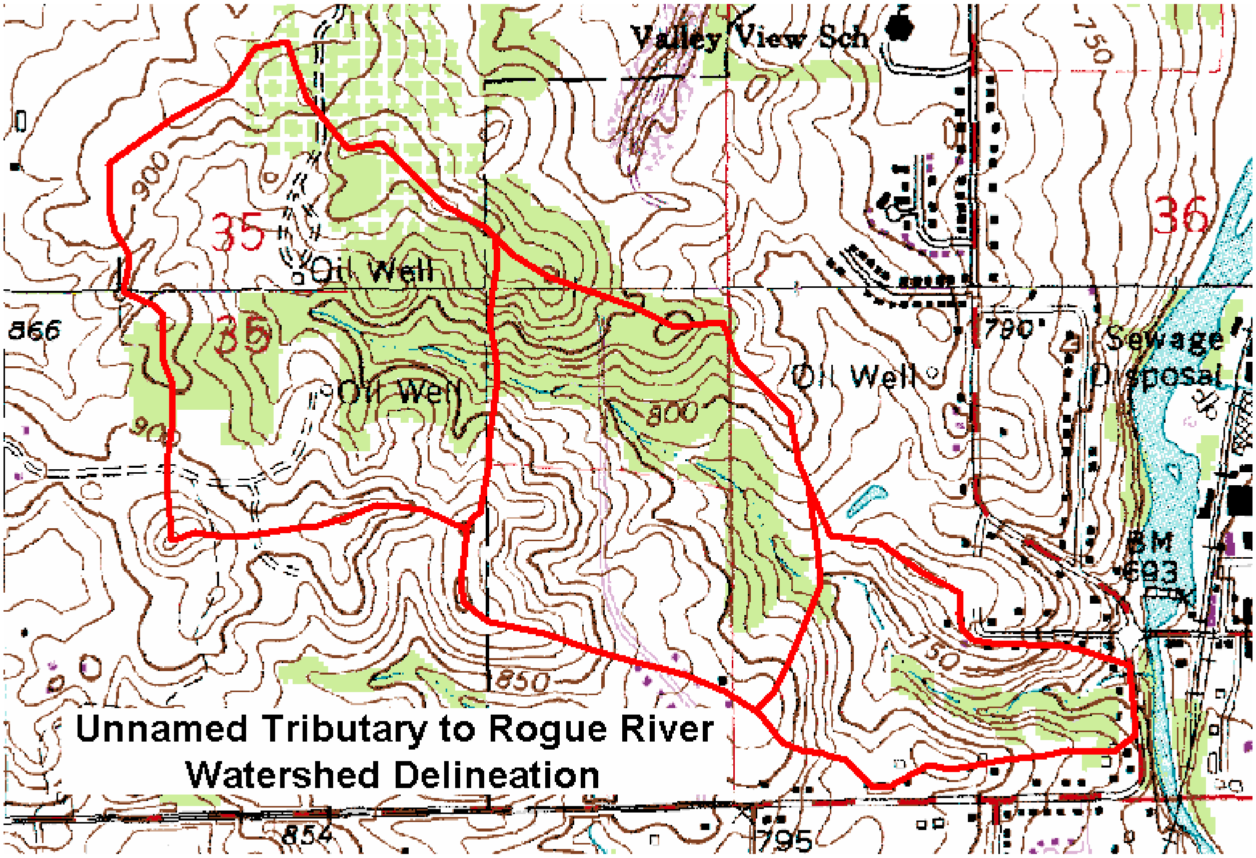

Figure 1: Delineated Watershed

Figure 2: July 1992 Aerial Photo with watershed delineation (red line) superimposed

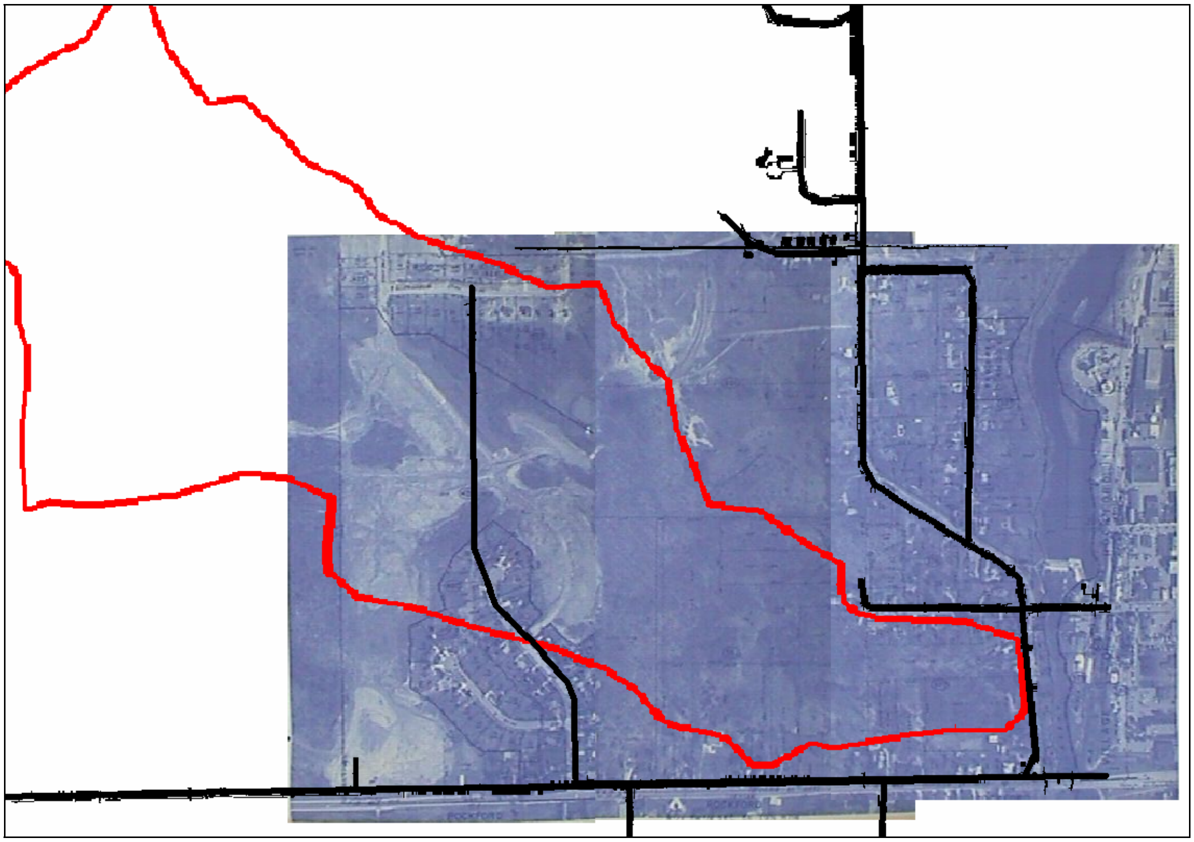

Figure 3: Spring 1998 aerial photos with watershed delineation (red line) superimposed

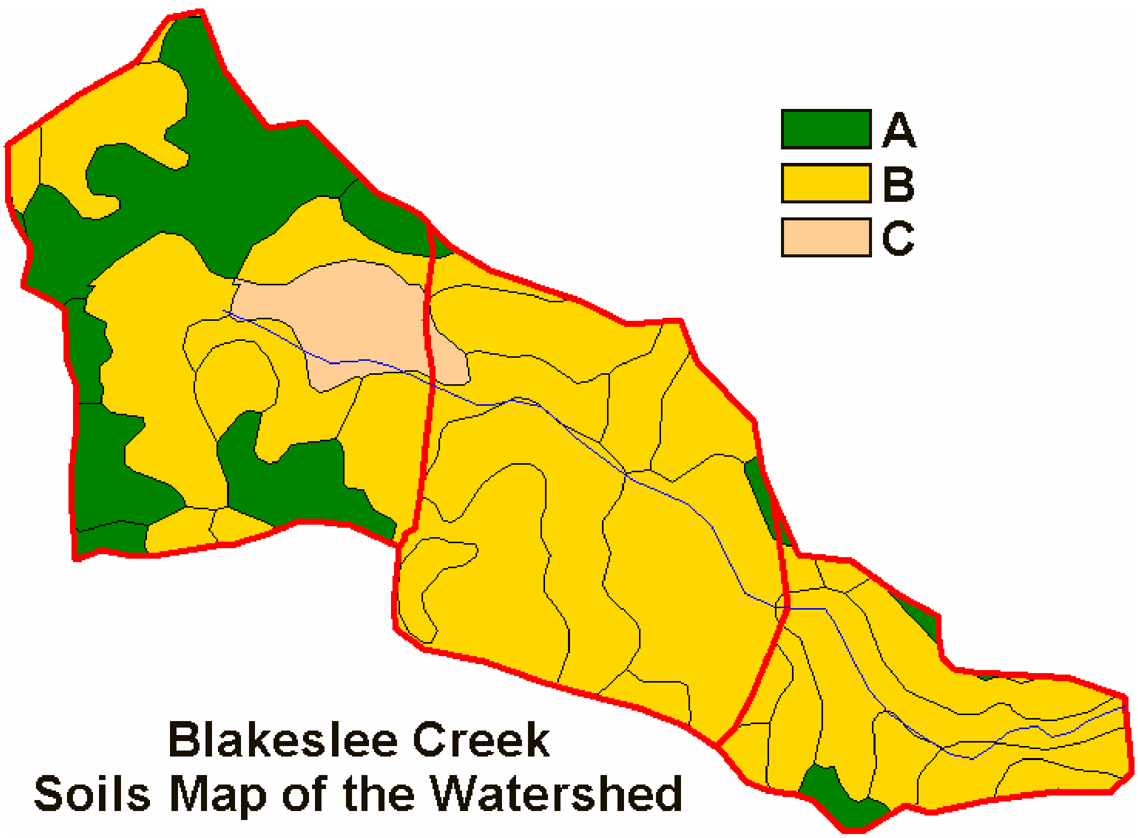

Figure 4 -Soils Map of Watershed

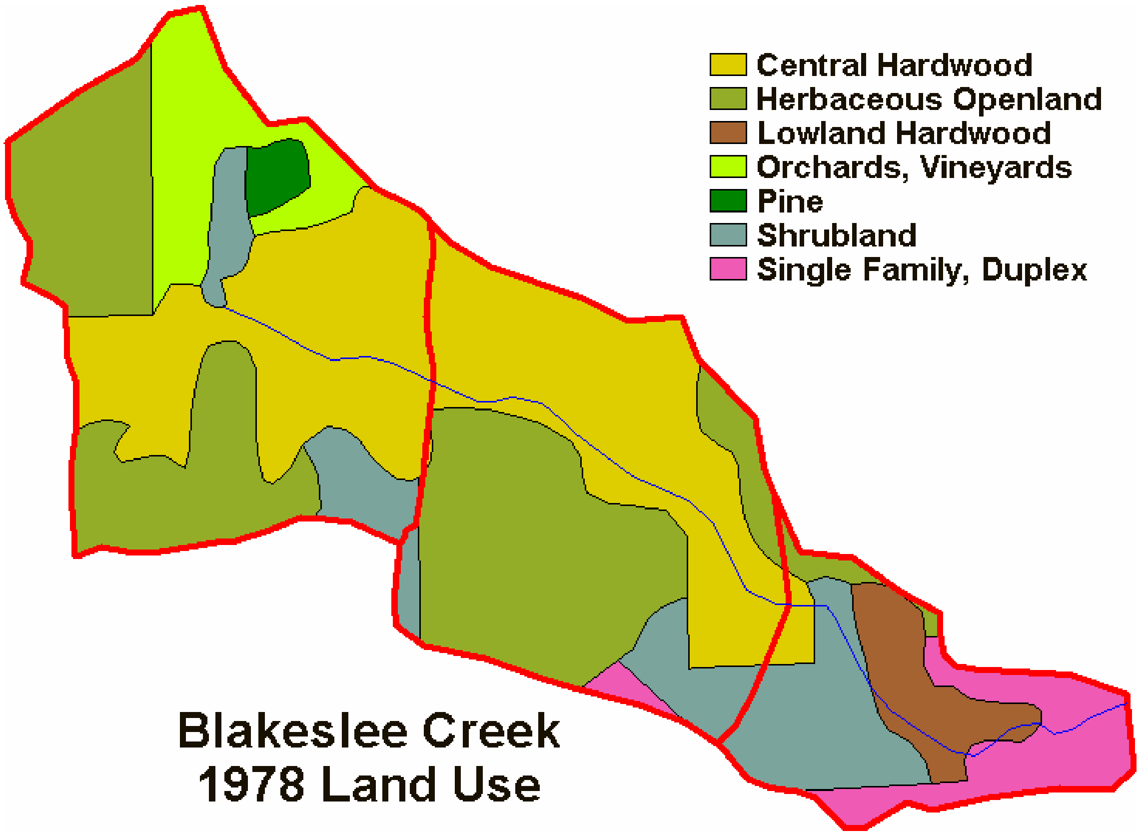

Figure 5 - 1978 Land Use

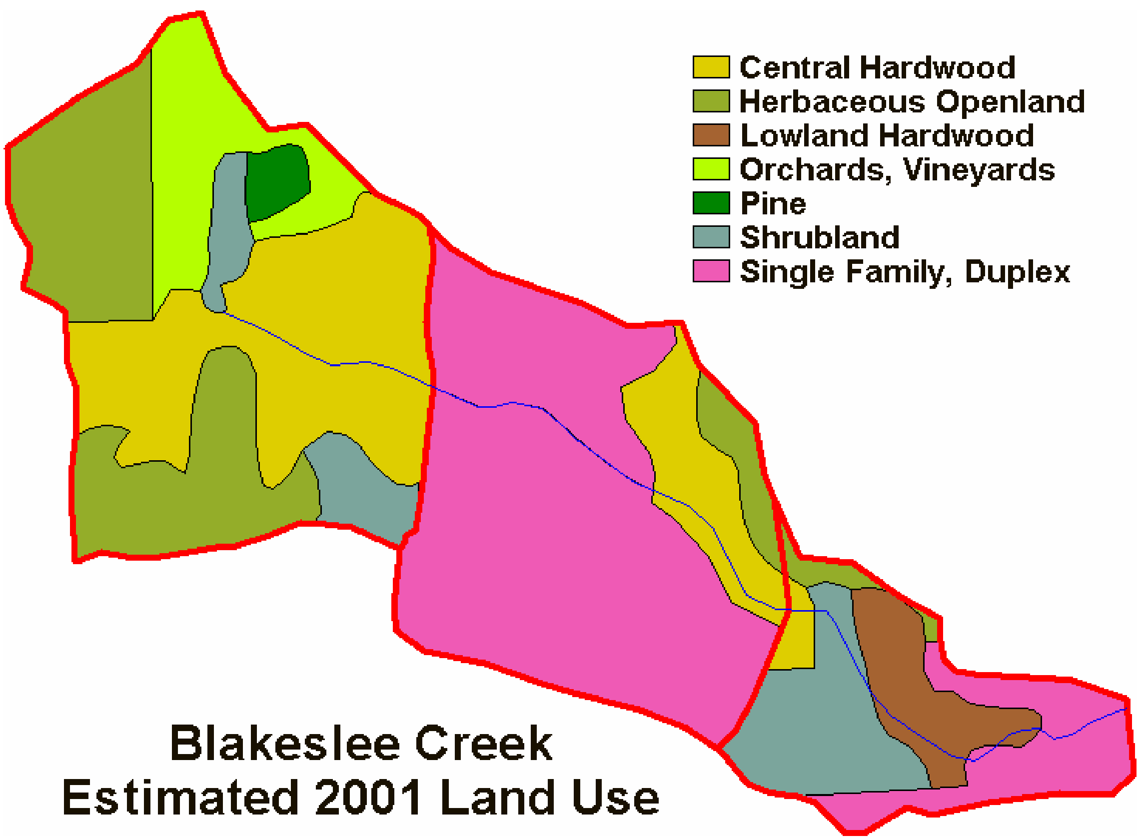

Figure 6 - Estimated Current Land Use

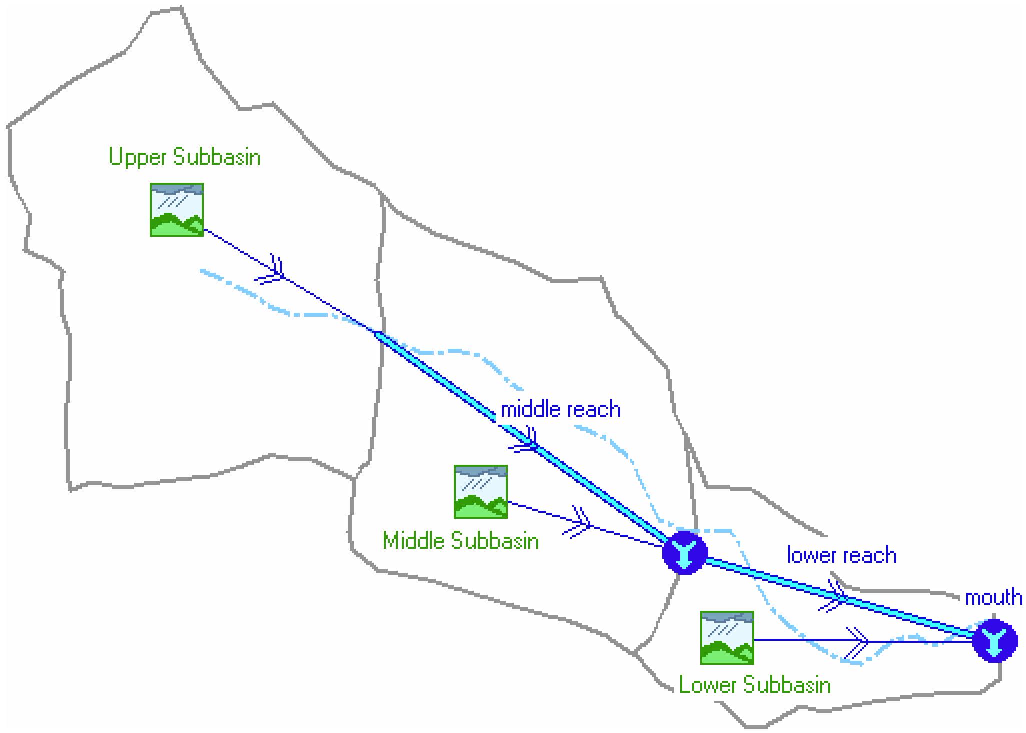

Figure 7 - Model Hydrologic Elements, 1978 land use

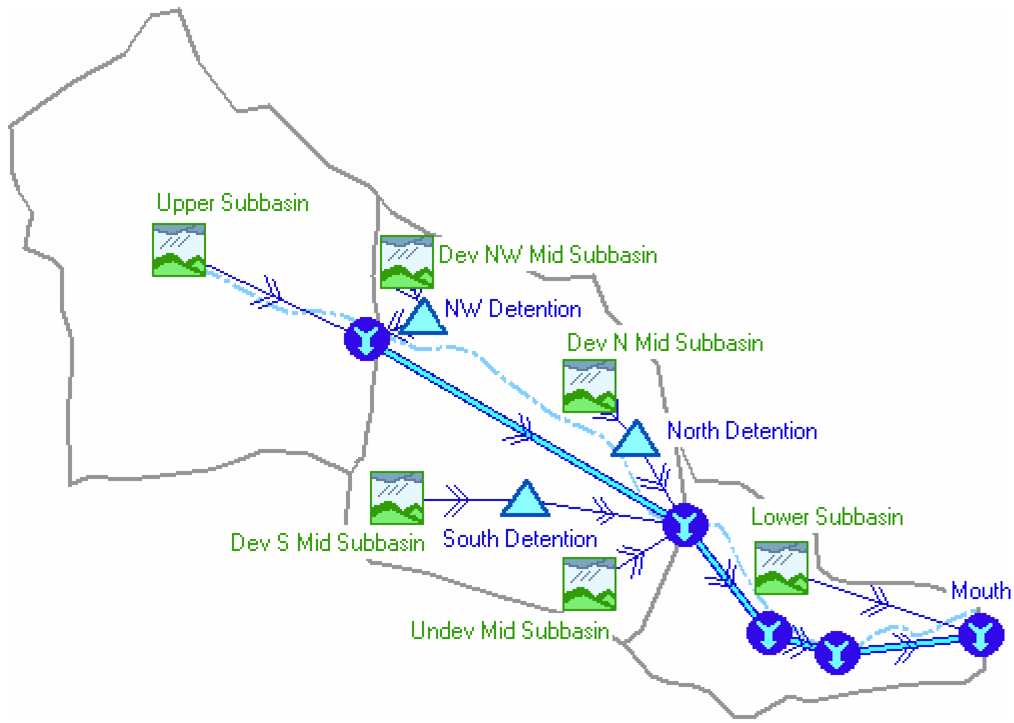

Figure 8 - Model Hydrologic Elements, current land use

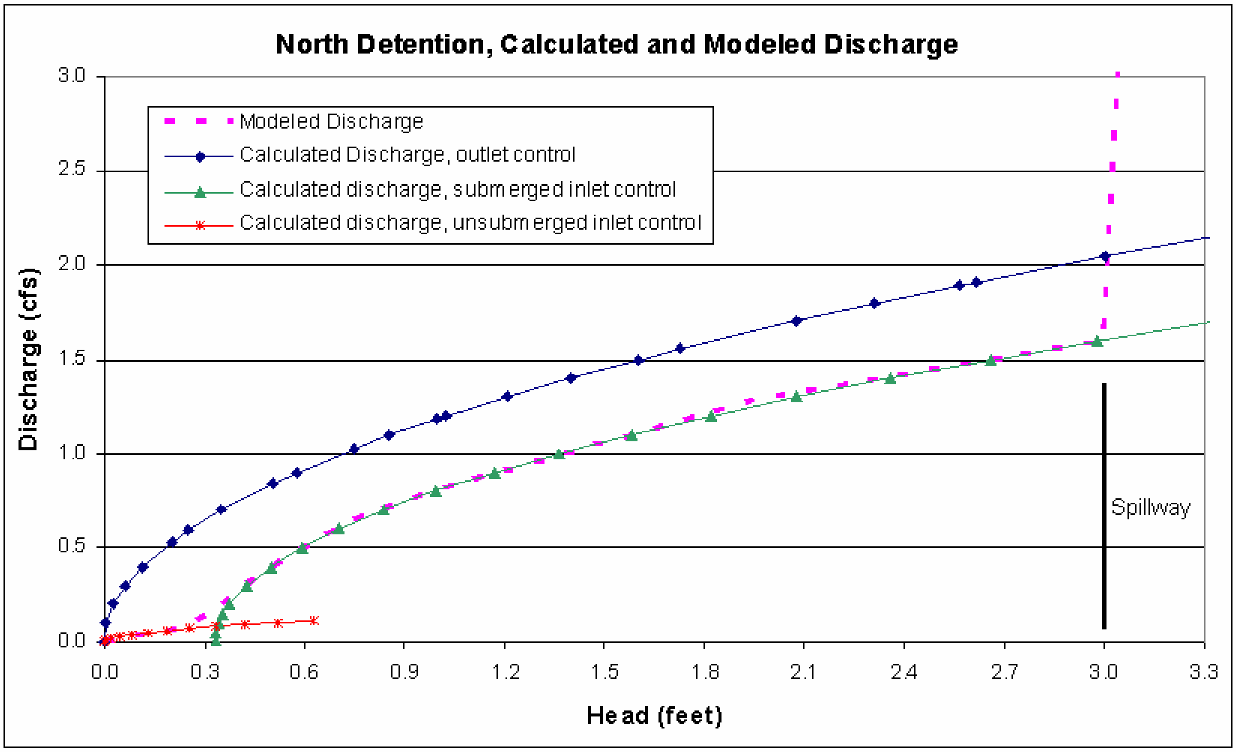

Figure 9: Calculated and Modeled North Detention Discharge

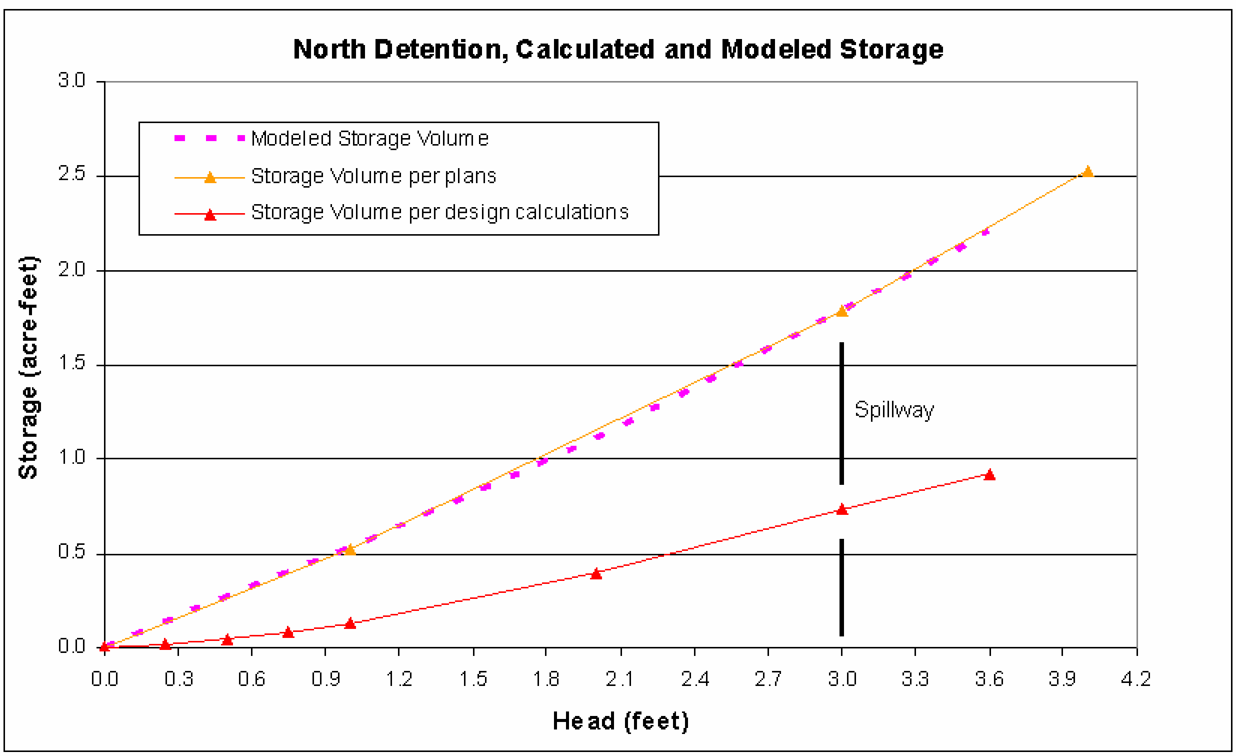

Figure 10: Calculated and Modeled North Detention Storage

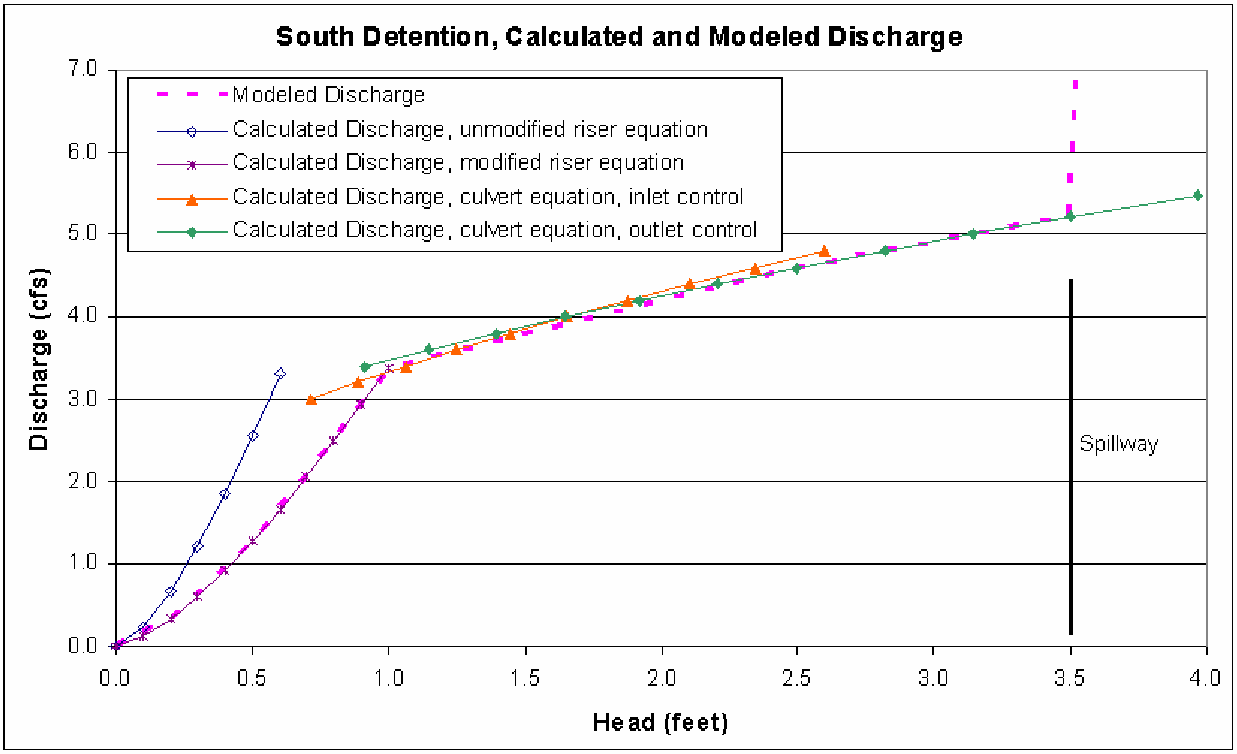

Figure 11: Calculated and Modeled South Detention Discharge

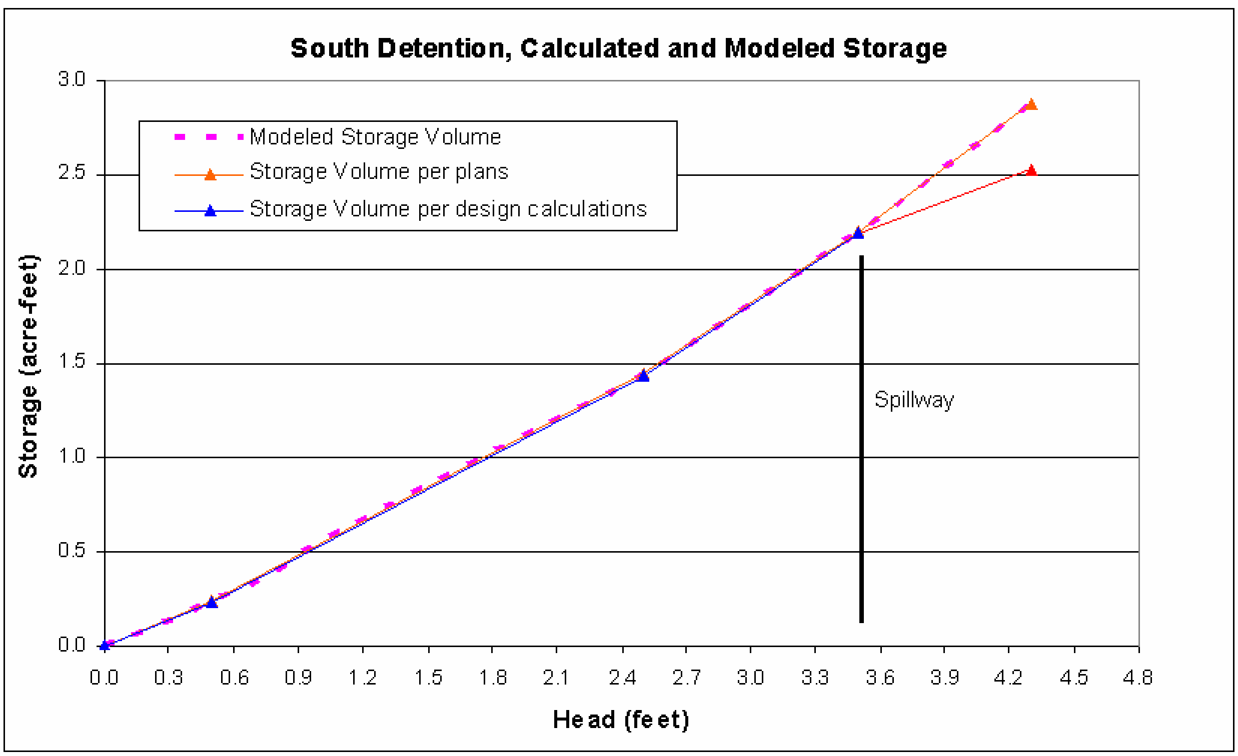

Figure 12: Calculated and Modeled South Detention Storage

Appendix B
Blakeslee Creek Hydrologic Model Calibration Technical Information

The monitoring data used to calibrate the model is shown in Figure 1. The rain that fell on May 14 and 15, 2001, was input into the model to establish antecedent moisture conditions. The actual calibration event occurred on May 16, 2001. In that event, 1.01 inches of rain fell in 7.5 hours, with 0.86 inches falling in 1.8 hours. This rain event has an expected average recurrence interval of approximately four months. This rainfall was selected because it is a continuous, higher-intensity rainfall. A storm with intermittent rainfall is a harder event to calibrate to. The rainfall also occurred during a time period when the gage located downstream of the subdivision was working. This location is of primary importance since the intended use of this model is to predict the changes in the stream's flow regime below the subdivision for the stream rehabilitation effort.

Upper Watershed Calibration

Based on the calibration data, the curve number for the upper watershed was lowered from 51 to 46.6. A before and after comparison of the hydrographs is shown in Figures 2 and 3.

Middle Watershed Calibration

Calibration of the middle watershed is more complex than the upper watershed because there are many more variables that can be adjusted. Figure 4, the pre-calibration hydrographs for the monitoring location downstream of the subdivision, indicated that the model needed to attenuate the peak flow from the upper subbasin because the modeled peak flow from the upper subbasin is roughly equal to the total monitored peak flow, which is not possible. Monitored flows also remain significantly elevated nine to eighteen hours after the rain event, which the model did not reproduce. Examining the detention monitoring data, Figures 6 and 7, indicated that the peak elevations from the detention ponds, especially the north detention pond, were too high, and that the modeled detention ponds drained too quickly.

To reduce peak flow through the reaches, the routing method was changed from lag, which does not provide any attenuation, to Modified Puls, which does. The values for the Modified Puls method were based on HEC-RAS modeling and adjusted, as necessary, based on the calibration data.

The initial modeling of the ponds used an outlet control equation. Further analysis revealed that the outlets for the detention ponds remain almost entirely under inlet control, and as a consequence, are more flow restrictive. This was particularly significant for the north detention pond. Adding this revision to the model would cause the ponds to drain more slowly, but further increase the maximum water elevations. As discussed in Appendix A, we discovered that the volume of the north detention pond is significantly larger when measured from the engineering plans, rather than the calculations provided by the consultant. Our field estimates of detention pond size more closely match the plan measurements. With these two changes, the modeled results for the north detention pond became reasonable.

The volumes of the south detention pond in the engineering calculations closely matches the volumes measured from the engineering plans, except for the volume above the spillway. Given this, the only way we could reasonably improve the fit of the model results to the monitoring data is to increase the storage coefficient, although we can see no reason for this particular drainage area to provide more storage prior to the detention pond than the other residential areas. When we doubled the storage coefficient, the fit only improved slightly, as shown in Figure 7. Since the model will be used to predict design flows from larger design storms, we prefer to maintain our standard relationship between time of concentration and storage coefficient. In order to get a better fit, we would have to significantly reduce either the drainage area or the curve number to reduce the volume of stormwater and further restrict the outlet, without having any physical basis for doing so. Alternatively, if the outlet were to become restricted during the storm event, that could cause the monitoring results we recorded, but would not be reproducible in the model. Because we cleared material from this outlet twice during the course of the study, we suggest that this is the more likely possibility.

The final hydrographs are shown in Figure 5. The model still over-predicts peak flow for the calibration storm event. Slightly reducing the curve number of the south subdivision subbasin would improve the modeled peak flows and elevations in both Figure 4 and 5. It is possible that the curve number is a little high, but houses are still being built and full build-out is likely in the very near future. We believe the calculated curve numbers are accurate for the full build-out. The predicted flows should reflect full build-out since the model will be used to provide design flows for stream rehabilitation BMPs. We did not reduce the curve numbers for the subdivisions.

Lower Watershed Calibration

Most of the changes between the pre-calibration and final hydrographs are due to changes in the upper watersheds and the change in reach routing method throughout the model. It was also necessary to change the time of concentration and storage coefficient for the lower subbasin to 1.5. This increase may reflect the importance of the wetland in attenuating and slowing smaller flows.

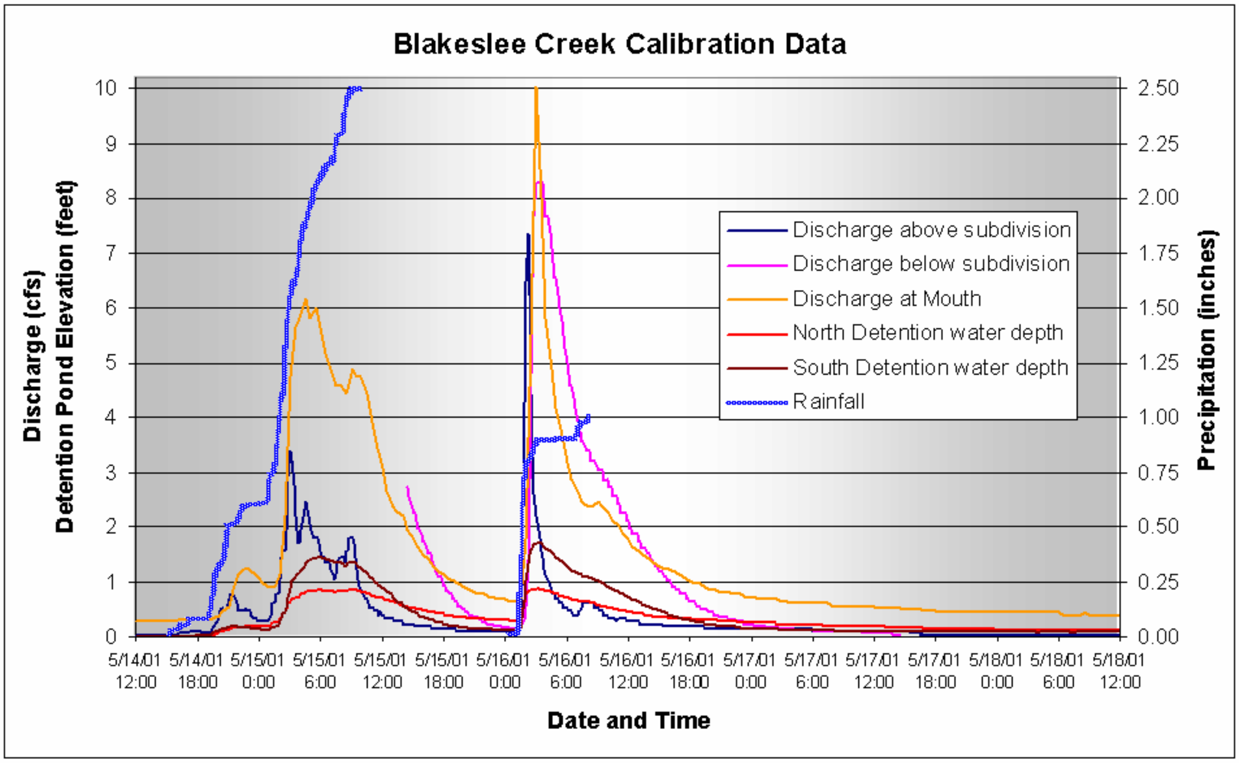

Figure 1: Monitoring data used to calibrate model

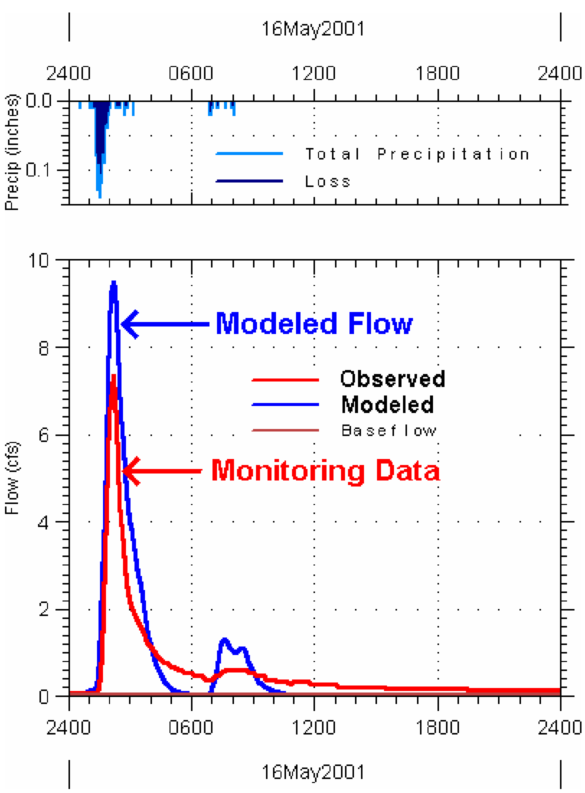

Figure 2: Upper Watershed, pre-calibration

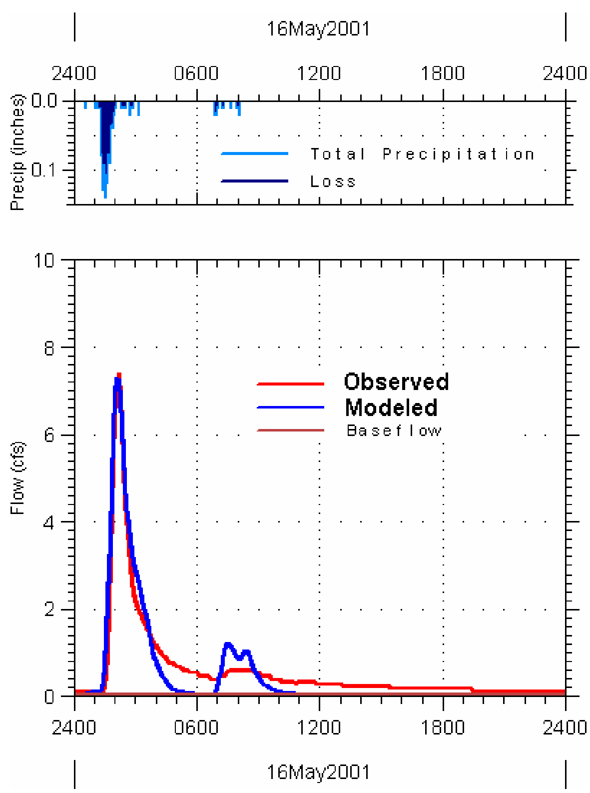

Figure 3: Upper Watershed, calibrated

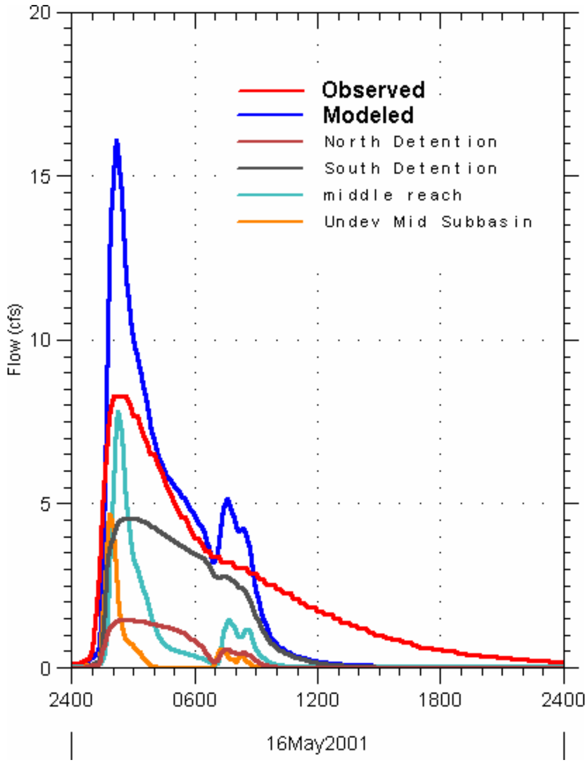

Figure 4: Middle Watershed, pre-calibration

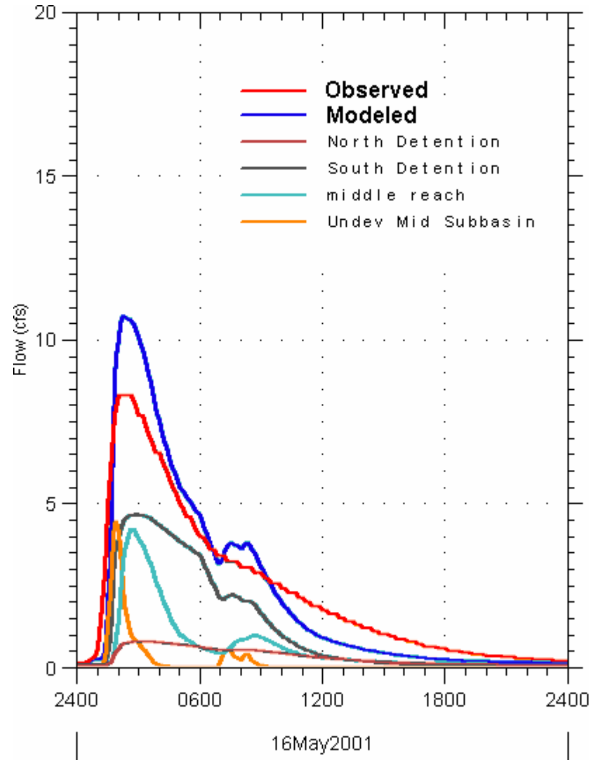

Figure 5: Middle Watershed, final

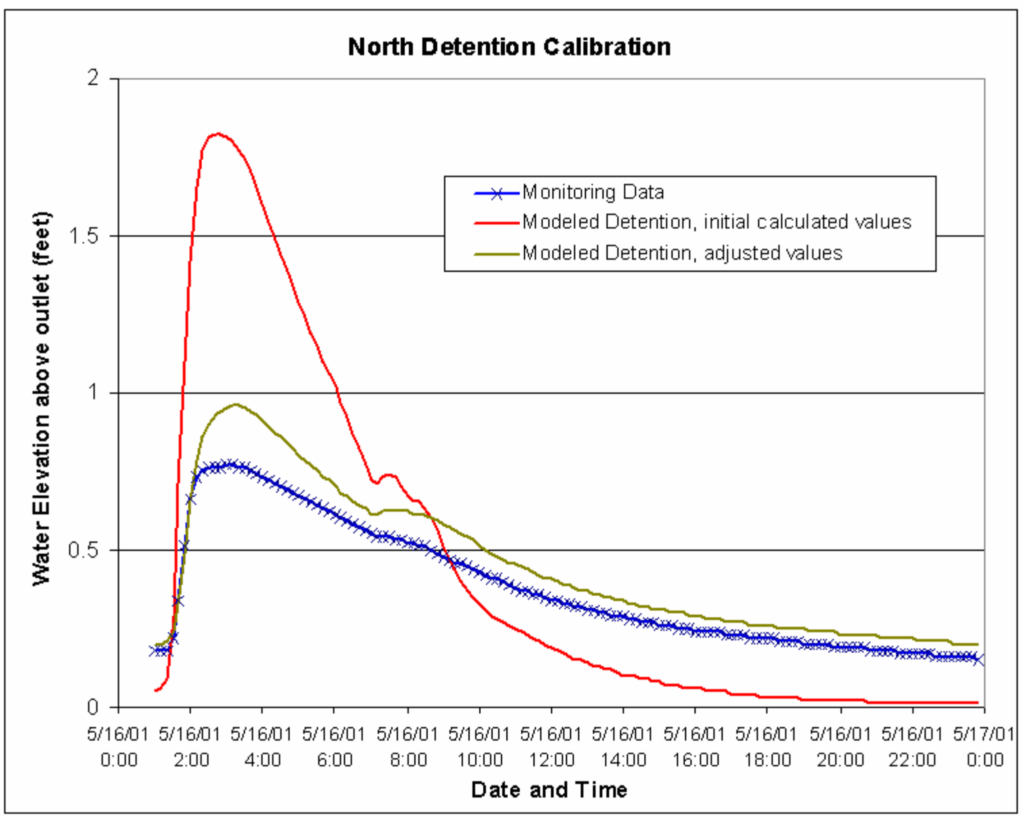

Figure 6: North Detention Calibration

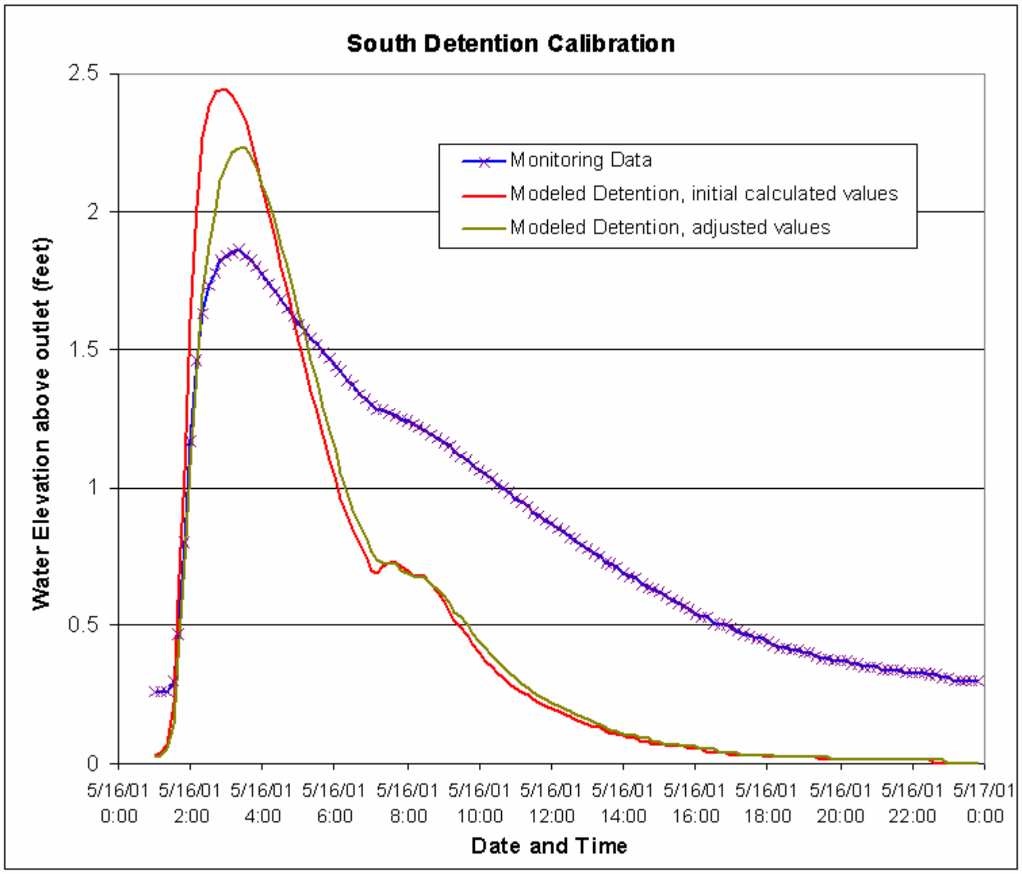

Figure 7: South Detention Calibration

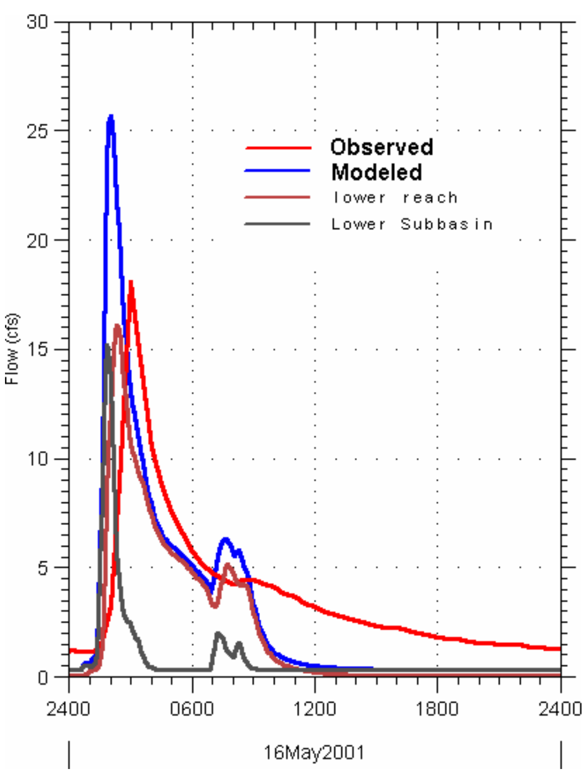

Figure 8: Lower Watershed, pre-calibration

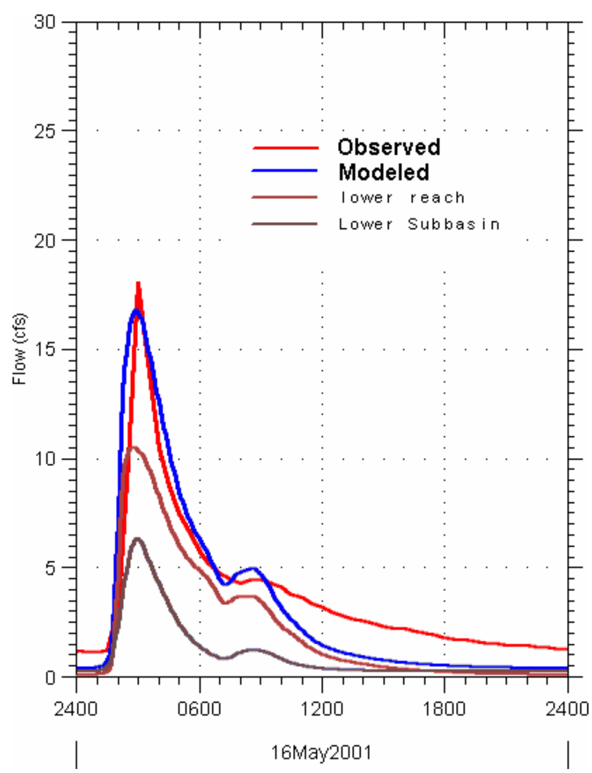

Figure 9: Lower Watershed, final

Appendix C
Blakeslee Creek Hydrologic Model Parameters

This appendix is provided so that the model could be recreated by a engineering consultant, or others, if desired. Table 1 provides the design rainfall values specific to the region of the state where Blakeslee Creek is located. Table 2 provides the parameters that were used in the hydrologic elements specified in HEC-HMS. Tables 3 through 5 provide the storage-discharge relationships used to model the detention ponds in the current land use scenario. Table 6 provides the reach parameters for the Modified Puls routing method.

Table 1: Design Rainfall Values for Kent County (Region 8)

Rainfall Duration	**Rainfall (inches) for given recurrence interval**			
	2-year	**10-year**	**25-year**	**100-year**
24-hour	2.37	3.52	4.45	6.15
12-hour	2.06	3.06	3.87	5.35
6-hour	1.78	2.64	3.34	4.61
3-hour	1.52	2.25	2.85	3.94
2-hour	1.37	2.04	2.58	3.57
1-hour	1.11	1.65	2.09	2.89
15-minute	0.64	0.95	1.20	1.66
5-minute	0.28	0.42	0.53	0.74

Table 2: Subbasin Parameters

Subbasin	**Area**		**Curve Number**		**Time of Concentration**		**Storage Coefficient**		**Baseflow Contribution (cfs)**
	Acres	**Square Miles**	**1978**	**Current**	**1978 (hours)**	**Current (hours)**	**1978**	**Current**	
Upper	92.72	0.145	46.6	46.6	0.52	0.52	0.52	0.52	0.04
Middle									
• Northwest residential	4.60	0.00719		70		0.20		0.20	0
• North residential	12.48	0.0195		70		0.30		0.30	0
• South residential	45.15	0.0778		70		0.30		0.30	0
• Undeveloped	13.89	0.0217		58		0.30		0.30	0
Overall	76.2	0.126	59	68*	0.30		0.30		
Lower	37.24	0.058	67	67	1.5	1.5	1.5	1.5	0.29

**The middle subbasin is divided into four drainage areas to represent current conditions. The curve number of 68 is a composite curve number that is listed for comparison, but is not specifically a model parameter.*

Table 3: North Detention Pond Parameters

Elevation (feet)	Storage (acre-feet)	Outflow (cfs)
0	0	0
0.25	0.131	0.07
0.5	0.263	0.40
0.75	0.394	0.65
1	0.525	0.80
2	1.100	1.30
3 (spillway elevation)	1.785	1.60
3.6 (maximum design flow over spillway)	2.210	20.30

Table 4: Northwest Detention Pond Parameters

Elevation (feet)	Storage (acre-feet)	Outflow (cfs)
0	0	0
0.25	0.048	0.03
0.5	0.097	0.15
0.75	0.145	0.24
1	0.193	0.29
2	0.405	0.48
3 (spillway elevation)	0.657	0.59
3.6 (maximum design flow over spillway)	0.813	7.47

Table 5: South Detention Pond Parameters

Elevation (feet)	Storage (acre-feet)	Outflow (cfs)
0	0	0
0.2	0.08	0.34
0.5	0.23	1.28
0.8	0.40	2.49
1	0.55	3.38
2.5	1.44	4.7
3.5 (spillway elevation)	2.19	5.48
4.3 (maximum design flow over spillway)	2.88	52.73
4.5 (top of berm)*	3.06*	65*
4.6*	3.20*	120*

**Values included because 1% storm exceeds capacity of detention pond.*

Table 6: Channel Reach Parameters

Reach Description	Storage (acre-feet)	Discharge (cfs)
Middle	0	0
	0.06	0.3
	0.13	1
	0.31	4
	0.57	10
	1.37	30
	1.87	40
Lower, upper section	0	0
	0.03	2.3
	0.04	`4
	0.06	8
	0.07	10
	0.11	20
	0.15	200
Lower, middle section	0	0
	0.002	0.11
	0.013	1.5
	0.034	3
	0.069	5
	0.138	10
	0.287	20
	0.459	200
Lower, lower section	0	0
	0.010	0.3
	0.043	2.2
	0.057	3.6
	0.070	5
	0.084	7
	0.095	9
	0.150	20
	0.210	200